

RiverFlood-1D v 1.0: coupling parabolic and backwater (PAB) scheme and Muskingum-Cunge model for river flood routing, with application to the Po River (Italy)

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ARTICLE INFO

Keywords:

Discharge routing
One-dimensional
Parabolic and backwater
Muskingum-Cunge
Arbitrary cross-section
downstream condition

ABSTRACT

RiverFlood-1D is a discharge routing model for simulating flood wave propagation in rivers with arbitrary cross-section geometries. It couples the parabolic-backwater (PAB) scheme with the Muskingum-Cunge (MC) approach, dynamically updating routing parameters at each computational step based on steady water surface profiles that account for downstream conditions. The model was verified against four test cases, including compound channels, mixed flow regimes, and a dam-break experiment with highly irregular cross sections. It was then applied to a real flood event along a 60 km reach of the Po river (Italy, April 2009), achieving mean absolute percentage errors of 0.230 and 0.119 for discharge and water depth, respectively, comparable to HEC-RAS (0.210 and 0.080). RiverFlood-1D remained stable in the dam-break test where HEC-RAS's unsteady solver failed to converge. The model combines accuracy similar to full dynamic-wave solvers with greater numerical robustness, making it well suited for operational, real-time flood-forecasting applications.

1. Introduction

Flood routing models are widely used to predict the temporal and spatial evolution of flood hydrographs in open-channel systems. They support flood forecasting, flood-risk assessment, hydraulic design, reservoir operation, and emergency management by estimating how flood waves propagate, attenuate, and transform along river reaches. Two broad families of flood routing approaches are commonly recognized: hydraulic routing and hydrologic routing. Hydraulic routing techniques are based on the principle of mass and momentum conservation through the solution of the hyperbolic partial differential equations of unsteady open-channel flow, referred to as the Saint-Venant equations (Chaudry, 1993; Graf, 1998). Hydrologic routing models, by contrast, are based on the solution of the continuity equation and a conceptual relationship between the storage within the reach and discharge at the outlet. Typically, in watershed simulation studies, hydrologic routing is used on a reach-by-reach basis following the topological order of the drainage network from upstream to downstream (Rabuffetti et al., 2008; Amengual et al., 2017).

For real-time flood forecasting, hydrologic routing models are

attractive because of their simplicity, numerical robustness, stability and low computational demand (Lamberti and Pilati, 1996). However, classical hydrological routing models are generally designed to predict discharge propagation rather than water surface elevations, and they often require simplified representations of channel geometry (Salvati et al., 2024). As a result, their ability to represent compound cross-sections, floodplain channel interactions, backwater effects, and discontinuities in the water surface due to irregularity of the cross-section and riverbed geometry remains limited (Ferrick, 1985; Perumal and Sahoo, 2007, 2009). These limitations become critical in natural rivers characterized by non-prismatic geometry, mixed flow regimes, or strong downstream boundary effects.

Among hydrological routing models, the Muskingum and Muskingum-Cunge (MC) approaches are widely applied both in scientific and engineering communities (Ceppi et al., 2013; Amengual et al., 2017; Barbetta et al., 2017; Franchini et al., 2011). The Muskingum method originally proposed by McCarthy (1938, 1940), was subsequently improved by Cunge (1969) by matching the numerical diffusion of the finite difference representation of the kinematic wave with the physical diffusion. The widespread use of MC method motivated many

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<https://doi.org/10.1016/j.envsoft.2026.107104>

Received 27 September 2025; Received in revised form 16 June 2026; Accepted 4 July 2026

Available online 6 July 2026

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authors to introduce further improvements, such as the updating of parameters values as a function of a reference actual discharge, giving birth to the variable parameter MC method (Ponce and Yevjevich, 1978; Ponce and Theurer, 1982; Ponce and Chaganti, 1994).

Recent studies have investigated the accuracy and the mass conservation of the variable parameters MC method, and hydrologic routing models in general. Cappelaere (1997) provided a derivation of a discharge advection-diffusion type equation to consider the influence of pressure gradients in the MC method. Todini (2007) revised the derivation of the MC equation proposing a new algorithm, the Muskingum-Cunge-Todini, that solved the mass balance error and steady state inconsistencies. Price (2009) deduced the nonlinear Muskingum and diffusion wave methods for flood routing from a conservative modified-advection equation as an approximation of the one-dimensional Saint-Venant equations. Moreover, he showed that the equations of Cappelaere (1997) and Todini (2007) are solutions of the proposed method.

When high accuracy in flood routing and water surface elevation prediction is needed for complex cases, a solution of the full Saint-Venant equations is required. The momentum equation includes the local and convective inertial terms, pressure gradient, gravity effect, and friction losses (Chow et al., 1988). Sophisticated numerical methods solving the full equations can take into account flow field and riverbed discontinuities (Cimorelli et al., 2012), or to consider wave propagation on dry bed for dam-break analysis (Cozzolino et al., 2012). Removing some terms of the momentum conservation equation leads to the diffusive, or parabolic, wave equation (Zoppou, 2001) and kinematic, or elliptic, wave equation (Tseng, 2010). The underlying simplification of the diffusion method is the neglect of the inertial terms (local accelerations, momentum fluxes) with respect to other terms (bed slope, depth variation, friction). This approximation has proven to be adequately accurate under various open-channel flow conditions, especially in rivers and natural streams (Ponce and Yevjevich, 1978; Cappelaere, 1997).

In recent years, data-driven and hybrid approaches have increasingly complemented physically based flood models. Machine-learning surrogate models and emulators, such as Gaussian-process emulators of inundation outputs (Donnelly et al., 2022), physics-informed neural networks acting as surrogates of hydrodynamic simulators (Donnelly et al., 2024), and deep-learning predictors of the hydraulic response of infrastructure during extreme floods (Mehran Ahmadi et al., 2024), have been coupled with numerical solvers to cut the computational cost of repeated simulations, while advanced high-resolution numerical schemes continue to be developed for complex free-surface flows (Torabbeigi et al., 2024, 2026). These approaches are effective for emulation and rapid scenario screening, but they still depend on an underlying process-based model for training data and physical interpretability. Efficient and robust process-based routing schemes therefore remain essential, both as stand-alone forecasting tools and as the simulators that such surrogate models emulate.

Among parabolic-wave approaches, the Parabolic and Backwater model (PAB) proposed by Todini and Bossi (1986), represents an important development because it combines flood-wave propagation with the computation of a steady backwater profile. The method was derived from the analytical solution of the linearized convective-diffusive equation (Hayami, 1951; Dooge, 1973). It is based on the successive application of locally linearized parabolic equations in integral form and computation of the steady-state backwater profile. Initially restricted to subcritical flow conditions in single open-channels, a modified version of the PAB model, called Linearized Parabolic Model (LPM), was presented by Della Morte et al. (2001), to consider channel networks. A further improved version of PAB was presented by Cimorelli et al. (2013) who developed the ILILPM model (Improved Locally and Instantaneously Linearized Parabolic Model) which can consider transcritical flow regime and transitions from free-surface to pressurized flow in tree-like channel networks.

Despite these developments, a methodological gap remains between classical hydrologic routing models and full dynamic-wave hydraulic solvers. Classical MC models are computationally efficient, stable, and suitable for operational forecasting, but they usually rely on simplified channel properties and have limited capacity to represent arbitrary compound cross-sections and downstream backwater effects. Full Saint-Venant solvers can represent these processes more explicitly, but they are computationally more demanding and may experience convergence difficulties in highly irregular geometries, rapidly varying flows, mixed flow regimes, or wetting-drying conditions. This gap motivates the development of an intermediate modelling approach that retains the efficiency and robustness of hydrologic routing while incorporating a more physically consistent representation of cross-section geometry and downstream hydraulic control. Furthermore, simplified 1D hydraulic models are particularly valuable for uncertainty analysis in flood risk applications: Bozzi et al. (2015) demonstrated, using both analytical derived-distribution methods and Monte Carlo simulations on the Po and Garonne rivers, that 1D steady-flow models are well suited to propagate roughness and discharge uncertainties into water level predictions, and that such uncertainties can produce water level errors exceeding 1 m, underscoring the practical importance of retaining physically meaningful yet computationally efficient 1D formulations in operational forecasting chains. This study addresses this gap by presenting RiverFlood-1D, a physically based discharge routing model for unsteady open-channel flow in arbitrary cross-section geometries and mixed flow regimes. The model couples the PAB scheme with the MC discharge routing approach. The methodological novelty is that the MC routing parameters are dynamically evaluated at each time step on the bases of a steady water surface profile in an arbitrary cross-section geometry that accounts for downstream conditions. The model can be seen as a generalization of the MC flood routing approach to compound arbitrary cross section with downstream effect. The specific contributions of this work are therefore: (i) the first formulation in which the MC parameters are evaluated, at every time step, from a steady mixed-regime water-surface profile in arbitrary compound cross-sections that explicitly accounts for the downstream boundary condition; (ii) an open-source implementation, RiverFlood-1D, that reads standard HEC-RAS geometry files and runs without case-specific numerical tuning; and (iii) the demonstration that this scheme retains accuracy comparable to a full dynamic-wave solver while remaining stable in geometries where the latter fails to converge. The aim is not to surpass full hydraulic solvers in accuracy, but to provide a robust, computationally efficient and easily coupled alternative for real-time flood forecasting and critical applications where efficiency is crucial.

The model was compared with four benchmark test cases: steady water surface profile in a compound channel measured in laboratory experiment, steady and unsteady analysis in a synthetic compound channel with mixed flow regime, and water surface profile of a laboratory dam break experiment over an initially dry bed with highly irregular cross sections. Finally, the model was applied to the simulation of a real flood event that occurred in April 2009 on the Po River, Italy.

2. Materials and methods

2.1. Theoretical background

2.1.1. The Saint-Venant equations

One-dimensional unsteady flow in open-channel is described by the Saint-Venant equations consisting of the continuity equation, Eq. (1), and the momentum equation, Eq. (2):

$$\frac{\partial Q}{\partial s} + \frac{\partial A}{\partial t} = 0 \quad (1)$$

$$\frac{1}{A} \frac{\partial Q}{\partial t} + \frac{1}{A} \frac{\partial}{\partial s} \left(\frac{Q^2}{A} \right) + g \frac{\partial y}{\partial s} - g(S_0 - S_f) = 0 \quad (2)$$

where Q is the flow rate [L^3T^{-1}], A is the cross-sectional area [L^2], s is the distance along the channel [L], y is the flow depth [L], g is the acceleration due to gravity [LT^{-2}], S_0 is the channel bed slope [LL^{-1}], and S_f is the friction slope [LL^{-1}]. The momentum equation consists of several terms: the local acceleration term (first term), that describes the change in momentum due to the change of velocity over time; the convective acceleration term (second term), that represents the variation in momentum resulting from changes in velocity along the channel; the pressure force term (third term) indicates a force that is proportional to the change in water depth along the channel; the gravity force term (fourth term) denotes a force proportional to the bed slope; the friction force term (fifth term) denotes a force proportional to the friction slope. The Saint-Venant equations have various simplified forms depending on the accuracy desired. The simplest equation is the kinematic wave model which neglects the local acceleration, convective acceleration, and pressure terms. The diffusion wave model neglects the local and convective acceleration terms. The dynamic wave model considers all the acceleration and the pressure terms.

2.1.2. The Muskingum-cunge routing scheme

The MC routing scheme is based on the solution of mass conservation (Eq. (3)) and a relationship between storage, inflow and outflow in a channel reach (Eq. (4)):

$$\frac{dS}{dt} = I - O \quad (3)$$

$$S = k[xI + (1 - x)O] \quad (4)$$

where S is the volume stored in the reach [L^3], k is a storage coefficient [T], x is a weighting factor [-], I is the inflow discharge at the upstream section [L^3T^{-1}], and O is the outflow discharge at the downstream section [L^3T^{-1}]. The finite difference formulation of Equations (3) and (4) results in the MC equation (Cunge, 1969):

$$O_{t+\Delta t} = C_1 I_{t+\Delta t} + C_2 I_t + C_3 O_t \quad (5)$$

where:

$$C_1 = \frac{-2kx + \Delta t}{2k(1 - x) + \Delta t} \quad (6)$$

$$C_2 = \frac{2kx + \Delta t}{2k(1 - x) + \Delta t} \quad (7)$$

$$C_3 = \frac{2k(1 - x) - \Delta t}{2k(1 - x) + \Delta t} \quad (8)$$

in which Δt [T] is the routing time increment of the finite difference cell. In the original Muskingum equation, the storage coefficient k and the weighting factor x are obtained by calibration with observed hydrographs (Miller and Cunge, 1975). MC approach defines the coefficients k and x based on the characteristics of the flow and the channel (Cunge, 1969; Koussis, 1978; Ponce and Theurer, 1982) as:

$$k = \frac{\Delta s}{c} \quad (9)$$

$$x = \frac{1}{2} \left(1 - \frac{Q}{S_0 c B \Delta s} \right) \quad (10)$$

where Δs is the reach length [L], c is the flood wave celerity [LT^{-1}], and B is the top width [L]. Cunge (1969) demonstrated that MC scheme is equivalent to a diffusion wave model. To account for flood wave celerity nonlinearity, Weinmann and Laurenson (1979), and Ponce and Yevjevich (1978) presented the concept of variable coefficients by recomputing coefficients k and x for every time step and reach as a function of updated values of discharge and wave celerity. In addition,

they proposed the following expressions for C_1 , C_2 , and C_3 :

$$C_1 = \frac{-1 + C + D}{1 + C + D} \quad (11)$$

$$C_2 = \frac{1 + C - D}{1 + C + D} \quad (12)$$

$$C_3 = \frac{1 - C + D}{1 + C + D} \quad (13)$$

derived in terms of the dimensionless Courant number (C), and cell Reynolds number (D), which is the ratio of the physical and numerical diffusivities:

$$C = \frac{c \Delta t}{\Delta s} \quad (14)$$

$$D = \frac{Q}{S_0 c B \Delta s} \quad (15)$$

An alternative expression for D is:

$$D = \frac{K}{\sqrt{S_f} c B \Delta s} \quad (16)$$

where K is the cross-section conveyance [L^3T^{-1}].

2.2. RiverFlood-1D model formulation

The development of the RiverFlood-1D model is based on the following assumptions.

- the inertial terms are negligible, hence unsteady flow is well represented by the diffusion wave model
- the diffusion wave model approximation of the Saint-Venant equations can be replaced by the MC routing scheme
- the flood wave slope cannot be neglected in comparison with bed slope
- the parameters of the MC routing scheme are updated at each time step on the bases of steady water surface profile in an arbitrary cross section geometry that accounts for downstream conditions

The simulation algorithm is implemented as follows (Fig. 1).

1. Provide an initial solution at time $t = 0$ computed as a steady flow water surface profile for a given discharge.
2. Set flow rate at the upstream end of the reach from user specified discharge hydrograph.
3. Update the steady flow water surface profile.
4. For each cross-section compute top width and kinematic wave celerity as the derivative of the Manning (1891) equation:

$$c = \frac{dQ}{dA} = \frac{1}{B} \frac{\sqrt{S_f}}{n} \left(\frac{2}{3} A R^{-1/3} \frac{\Delta R}{\Delta y} + B R^{2/3} \right) \quad (17)$$

where n is the Manning roughness coefficient [$T/L^{1/3}$], R is the hydraulic radius [L], ΔR [L] is the variation of hydraulic radius computed for a decrease of water depth [L], Δy , of 0.001 m.

5. Route discharge from upstream to downstream with MC method
6. Update $t = t + \Delta t$ and restart from (2).

A steady water surface profile is computed assuming mixed, subcritical and supercritical, flow. To this purpose two boundary conditions are necessary, one at the upstream end and one at the downstream end of the reach. In RiverFlood-1D, three types of boundary conditions are implemented: normal depth, critical depth and a user assigned water depth.

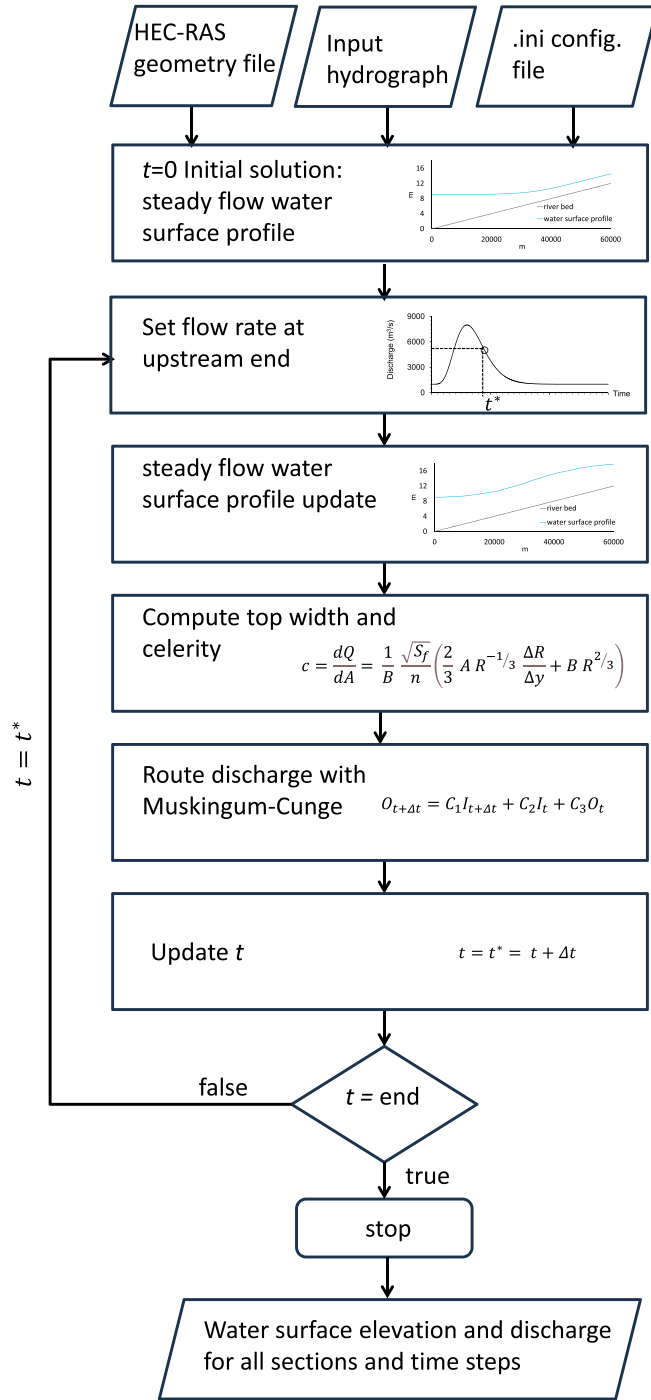


Fig. 1. RiverFlood-1D flowchart of simulation algorithm implementation.

2.2.1. Steady flow water surface profile computation

Algorithms for steady water surface profile computation are mostly inspired by HEC-RAS software (U.S. Army Corps of Engineers, 2010). The water surface profile is calculated between consecutive cross-sections (Fig. 2) by solving the energy equation:

$$z_2 + y_2 + \alpha_2 \frac{V_2^2}{2g} = z_1 + y_1 + \alpha_1 \frac{V_1^2}{2g} + h_t \quad (18)$$

in which z = bed elevation [L], V = velocity [LT^{-1}], h_t = total energy loss between sections 1 and 2 [L], and subscripts 1 and 2 denote sections 1 and 2, respectively.

Given a subdivision of a cross-section in N parts using vertical

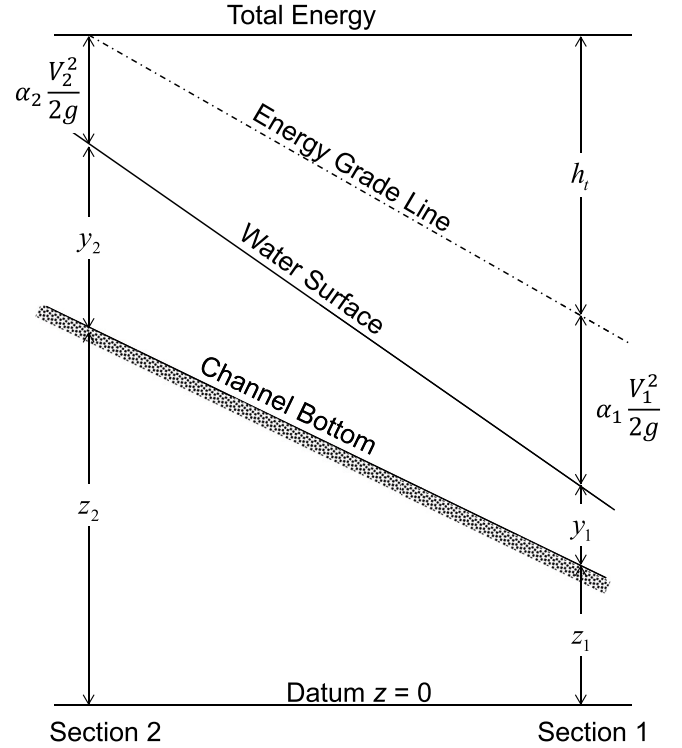


Fig. 2. Representation of terms in the energy equation for steady water surface profile computation.

bisecting lines as depicted in Fig. 3, the velocity distribution coefficient $[-]$, α , is computed as follows:

$$\alpha = \frac{A^2 \sum_{i=1}^N \left[\left(\frac{A_i R_i^{2/3}}{n_i} \right)^3 / A_i^2 \right]}{\left(\sum_{i=1}^N \frac{A_i R_i^{2/3}}{n_i} \right)^3} \quad (19)$$

where A_i , R_i , and n_i are wetted area [L^2], hydraulic radius [L] and Manning roughness coefficient [$\text{TL}^{-1/3}$], respectively, of the i_{th} subdivision.

The energy head loss, h_t , between two cross-sections is comprised of friction losses and contraction or expansion local losses:

$$h_t = LS_f + EC \left| \alpha_2 \frac{V_2^2}{2g} - \alpha_1 \frac{V_1^2}{2g} \right| \quad (20)$$

in which L = reach length [L]; EC = expansion or contraction loss coefficient $[-]$.

The friction slope at each cross-section is computed with the average conveyance equation:

$$S_f = \left(\frac{Q_1 + Q_2}{K_1 + K_2} \right)^2 \quad (21)$$

Two options are implemented in RiverFlood-1D for computing cross-section conveyance.

For a generic cross-section, conveyance is computed as:

$$K = \sum_{i=1}^N \frac{A_i R_i^{2/3}}{n_i} \quad (22)$$

If a significant portion of the cross-section has vertical slopes, the conveyance is computed as the sum of conveyance of the three subunits assigned by the user (Fig. 3), main channel, left overbank, and right overbank. Conveyance, K_j , of each of the three subunits is computed as

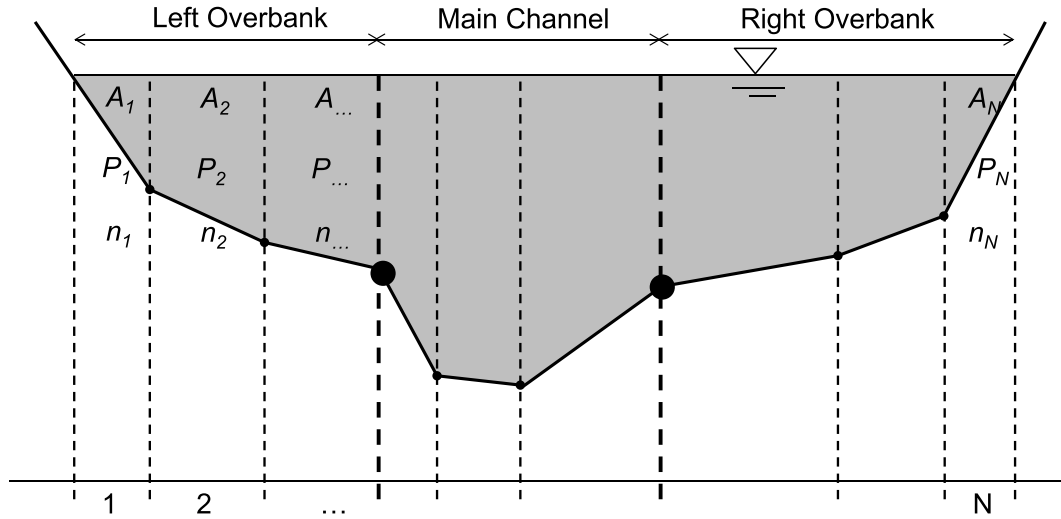


Fig. 3. Subdivision of a cross-section for discrete computation of hydraulic properties. A = wetted area, P = wetted perimeter, n = Manning roughness coefficient.

(Einstein and Banks, 1950):

$$K_j = \frac{A_j R_j^{2/3}}{n_j^{eq}} \quad (23)$$

where n^{eq} is the equivalent Manning roughness coefficient computed as:

$$n^{eq} = \left(\frac{\sum_{i=1}^M P_i n_i^{3/2}}{P} \right)^{2/3} \quad (24)$$

where M is the number of subdivisions within subunit, P_i and P are the wetted perimeter of the i_{th} subdivision and the subunit, respectively.

The unknown water surface elevation at a cross-section is computed by an iterative solution of Eqs. (18) and (20). The method for assuming water surface elevations in the iterative procedure varies from trial to trial. Details can be found in the HEC-RAS hydraulic reference manual (U.S. Army Corps of Engineers, 2024).

Critical depth, y_c , is computed by finding the root of equation (25), valid for any cross-section geometry, using Brent's method (Brent, 1973):

$$Q^2 B(y_c) - g A(y_c)^3 = 0 \quad (25)$$

The program does not assume a given flow regime for computing water surface profile (mixed flow regime). Determination of controlling flow regime as well as location of hydraulic jumps are based on specific force, SF [L^3], computed as follows:

$$SF = \frac{Q^2 \beta}{gA} + A y_g \quad (26)$$

where y_g = depth from the water surface to centroid of the area [L], and β = momentum coefficient [-] computed as follows:

$$\beta = \frac{A \sum_{i=1}^N \left[\left(\frac{A_i R_i^{2/3}}{n_i} \right)^2 / A_i \right]}{\left(\sum_{i=1}^N \frac{A_i R_i^{2/3}}{n_i} \right)^2} \quad (27)$$

The mixed flow regime calculation is described in the HEC-RAS hydraulic reference manual (U.S. Army Corps of Engineers, 2024).

2.3. Model implementation

RiverFlood-1D was developed in the Fortran 90 programming language due to its high portability and efficiency and to facilitate inclusion of the program in existing hydrological models mostly coded in Fortran language (Corbari et al., 2011; Ravazzani et al., 2011).

Selection of type of analysis, steady or unsteady, boundary conditions and simulation options are assigned by the user by a *INI* configuration file that consists of a text-based content with a structure and syntax comprising *key-value* pairs for properties. RiverFlood-1D reads HEC-RAS geometry files format (extension .gxx) for defining cross-section geometry and river schematic.

RiverFlood-1D conforms to European standards for writing and documenting exchangeable Fortran 90 code (Andrews et al., 2011) and FORD (2024), an automatic documentation generator for modern Fortran programs.

RiverFlood-1D is distributed under the GNU/GPL v3 license.

3. Results and discussion

3.1. Dataset, model setup and performance metrics

The model is tested against four benchmark cases: i) steady water surface profile in a laboratory test channel with floodplains, ii) steady water surface profile in a compound channel with mixed flow regime, iii) unsteady analysis in a compound channel with mixed flow regime, and iv) flood routing of a laboratory dam break experiment over an initially dry bed with highly irregular cross sections. Finally, the model is validated with the simulation of a real flood event that occurred in April 2009 on the Po River, Italy.

For each test, the model input consisted of cross-section geometry, Manning roughness coefficients, upstream discharge conditions, downstream water-level or normal-depth boundary conditions, and initial hydraulic conditions. The initial condition was obtained by computing a steady water-surface profile corresponding to the initial discharge. This profile was then used to initialize water depth, conveyance, celerity, and the MC routing parameters. Boundary conditions were imposed according to the hydraulic nature of each test: critical depth was used where the downstream control was governed by free outflow or critical-flow conditions, prescribed water-surface elevation was used for the synthetic mixed-regime channel, and normal depth was used for the Po River application. No calibration of Manning roughness was performed in the Po River case; roughness values were prescribed from previous hydraulic studies of the same river reach. The input datasets, model

parameters, and reference data used in each test are summarized in Table 1.

RiverFlood-1D results are compared to observations and HEC-RAS (release 6.0) simulation results, when available. Performance assessment has been undertaken with four indexes computed against the reference dataset, observations or HEC-RAS simulation results according to the test: the mean percentage error, MPE , the mean absolute percentage error, $MAPE$, the maximum absolute percentage error, $MaxPE$, and the root mean square error, $RMSE$, computed with the following equations.

$$MPE = \frac{1}{n} \sum_{i=1}^n \frac{x_i^s - x_i^r}{x_i^r} \quad (28)$$

$$MAPE = \frac{1}{n} \sum_{i=1}^n \left| \frac{x_i^s - x_i^r}{x_i^r} \right| \quad (29)$$

$$MaxAPE = \max_{i=1}^n \left| \frac{x_i^s - x_i^r}{x_i^r} \right| \quad (30)$$

$$RMSE = \left[\frac{\sum_{i=1}^n (x_i^s - x_i^r)^2}{n} \right]^{0.5} \quad (31)$$

where n is the number of data pairs, x_i^s is the i th simulated data, and x_i^r is the i th reference data.

3.2. Test 1: Steady water surface profile in a laboratory test channel with floodplains

The RiverFlood-1D model was tested by simulating a steady water surface profile in a channel with floodplains obtained in a laboratory test facility presented by Rashid and Chaudry (1995). The laboratory facility consisted of a flume of 21 m length, 0.93 m width and 0.56 m depth. The slope of the channel bottom was constant at 0.0021. The working section of the flume was 18.57 m long. The rectangular flume was converted into a channel composed by a main channel and floodplains (Fig. 4). The flow was critical at the downstream overflow section. Tests were conducted for various discharges. Details of the experiments may be found in Rashid (1991) and Rashid and Chaudry (1995).

For validation of RiverFlood-1D capabilities in simulating steady water surface profile in compound channels, the initial condition obtained with a constant discharge was compared to the water surface profile computed by RiverFlood-1D. In Fig. 5, the water surface profile computed by RiverFlood-1D is compared to the experimental observation and the water surface profile computed by HEC-RAS for two test cases: the first obtained with a discharge value that involved the floodplains (Fig. 5a) (Test 1 in Rashid and Chaudry, 1995) and the second obtained with a discharge that involved the main channel only (Fig. 5b) (Test 2 in Rashid and Chaudry, 1995). In both tests, a good agreement with the experimental and HEC-RAS computed water surface profile is achieved.

The comparison between the isolated main-channel and compound-channel simulations demonstrates that RiverFlood-1D accurately reproduces conveyance redistribution between the main channel and

floodplains without requiring geometric simplification. The agreement with both the laboratory observations and HEC-RAS therefore indicates that the backwater component of RiverFlood-1D provides a hydraulically consistent estimate of water level in compound sections before the model is applied to more complex mixed-regime and unsteady cases.

3.3. Test 2: Steady water surface profile in a compound channel with a mixed flow regime

The RiverFlood-1D model was tested by computing steady water surface profile in compound channel with a mixed flow regime. The test was adapted from problem 9-8, page 245, in Chow's "Open-Channel Hydraulics" (Chow, 1959). The geometry of the compound cross-section, Manning's roughness values, and channel bed profile are shown in Fig. 6.

The channel, with a total length of 3000 m, consists of three reaches of equal length of 1000 m and different bottom slopes in order to create a change in flow regime. Water surface elevation is set at 6 m at the downstream end while the upstream boundary condition was set to critical depth. In test case A, the steady water surface profile was computed for a discharge of 1000 m³/s that sets the water level within the main channel in most of the channel length. In test case B, the discharge was increased to 8000 m³/s to let the water surface profile enter the channel overbanks. The computed water surface profiles in comparison to the solution provided by HEC-RAS are shown in Fig. 7. Results show a substantial agreement between the two models. RiverFlood-1D performance indexes in simulating water depth, computed against HEC-RAS simulation results considered as the reference dataset are reported in Table 2. For the test case A, the average percentage error is low as 0.0003 with a maximum absolute percentage error of 0.008. Test case B shows slightly greater error values, mainly due to a different localization of the hydraulic jump, where the maximum absolute percentage error reaches 0.2206. This larger discrepancy is concentrated at the hydraulic jump: away from the regime-transition zone, the two profiles are practically indistinguishable ($MAPE = 0.0129$), and the maximum error reflects an essentially one-cell shift in the computed jump position rather than a systematic bias, as confirmed by the near-zero mean percentage error ($MPE = -0.0011$). This indicates that RiverFlood-1D resolves the backwater and overbank stages in a compound section as accurately as the full dynamic-wave solver, and that the main source of local difference is the representation of the jump as a discontinuity rather than a smoothed transition, an aspect examined further in Test 3.

3.4. Test 3: Unsteady flow in a compound channel with mixed regime

The RiverFlood-1D model was tested by simulating unsteady flow in a compound channel with mixed flow regime. The channel is the same as the one used in test 2.

The model is tested by routing two hypothetical hydrographs defined using a gamma function (Perumal and Ranga Raju, 2007):

$$I(t) = Q_b + (Q_p - Q_b) \left(\frac{t}{t_p} \right)^{1/(\gamma-1)} \exp \left[\frac{\left(1 - \frac{t}{t_p} \right)}{\gamma - 1} \right] \quad (32)$$

where $I(t)$ = inflow at time t , Q_b = base flow, Q_p = peak flow; t_p = time to

Table 1
Summary of datasets and parameters of tests.

Test	Reach length (m)	Number of cross sections	Slope	Manning channel/(bank) (s/m ^{1/3})	Time step number	Time step (s)	Data source
1	21	11	0.0021	0.013	-	-	Rashid and Chaudry (1995)
2	3000	61	0.0002-0.01	0.033/0.1	-	-	Chow (1959)
3	3000	61	0.0002-0.01	0.033/0.1	3000	60	Chow (1959)
4	53.6	63	0.002-1.4	0.0162	180	1	Caleffi et al. (2003)
5	58700	79	0.00002-0.012	0.03/0.09	526	1800	ARPA Piemonte

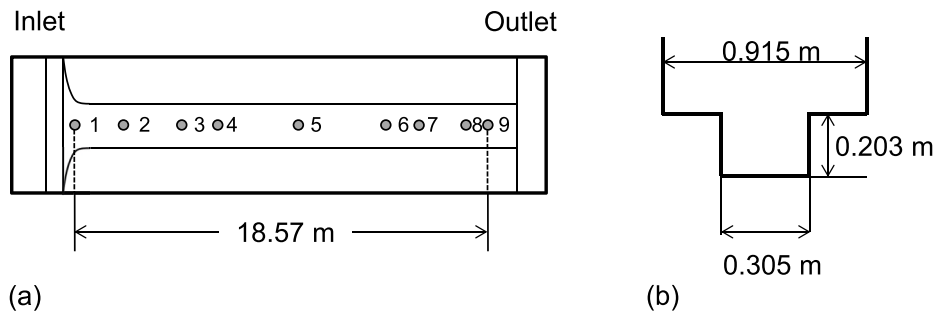


Fig. 4. Schema of the compound channel laboratory experimental apparatus: (a) longitudinal section with nine data measurements; (b) cross-section of the channel. (Adapted from Rashid and Chaudry, 1995).

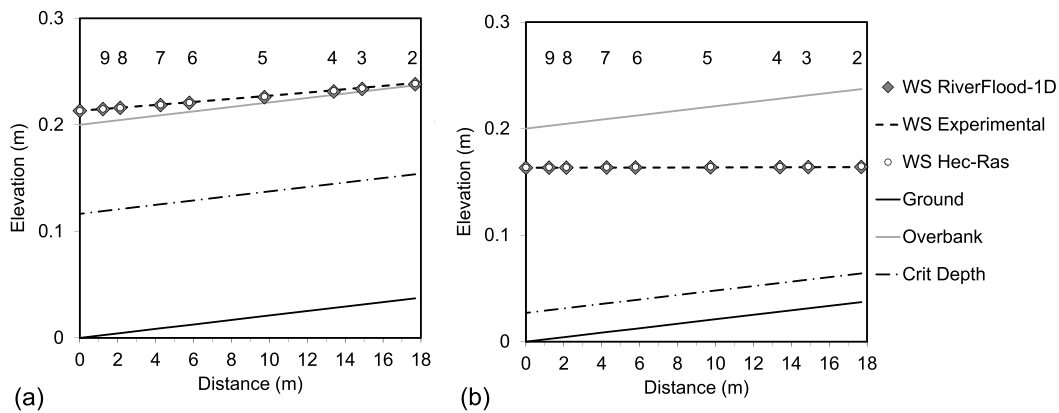


Fig. 5. Water surface profile (WS) computed by RiverFlood-1D, HEC-RAS and experimental observation for two test cases: (a) the first obtained with a discharge value that involved the flood plains, and (b) the second obtained with a discharge that involved the main channel only. Critical depth, ground and overbank elevation are shown. Flow direction is from right to left.

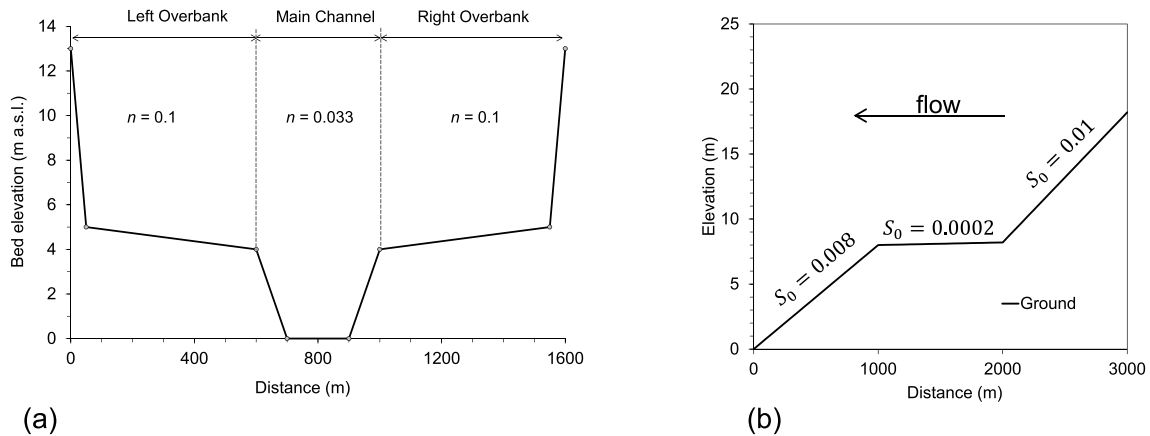


Fig. 6. (a) Cross section geometry and (b) channel bed profile of the compound channel used to test capability of the RiverFlood-1D model to deal with mixed flow regime.

peak, and γ is the skewness factor.

Two hydrographs that differ in shape are employed at the upstream end, with discharge in the range of the minimum and maximum values used in test 2 (Fig. 8). The inflow hydrograph parameters are given in Table 3. Water surface elevation is set at 6 m at the downstream end.

The computed discharge and water surface elevation along the channel for given times are shown in Fig. 9. Both RiverFlood-1D and HEC-RAS were able to find the solution of the mixed flow problem without any instabilities. Test 3A hydrograph is characterized by a small variation of discharge value in time. As a consequence, the resulting discharge along the channel is quite constant independently of the time

considered. On the contrary, the test 3B hydrograph shape has a significant variation of discharge along the channel for a given time. At $t = 10$ h, the hydrograph is in the rising limb, the discharge in downstream sections is lower than in the upstream ones. After the peak, at $t = 11$ h, discharge along the channel behaves in the opposite way, i.e., discharge is higher in downstream sections. In both test cases and at all times, the discharge along the channel computed by RiverFlood-1D is consistent with the results of HEC-RAS. Differences between the two models are evident in the water surface profile when a change of flow regime is achieved. HEC-RAS shows a smooth transition between supercritical and subcritical regimes (and vice versa) that occurs in the

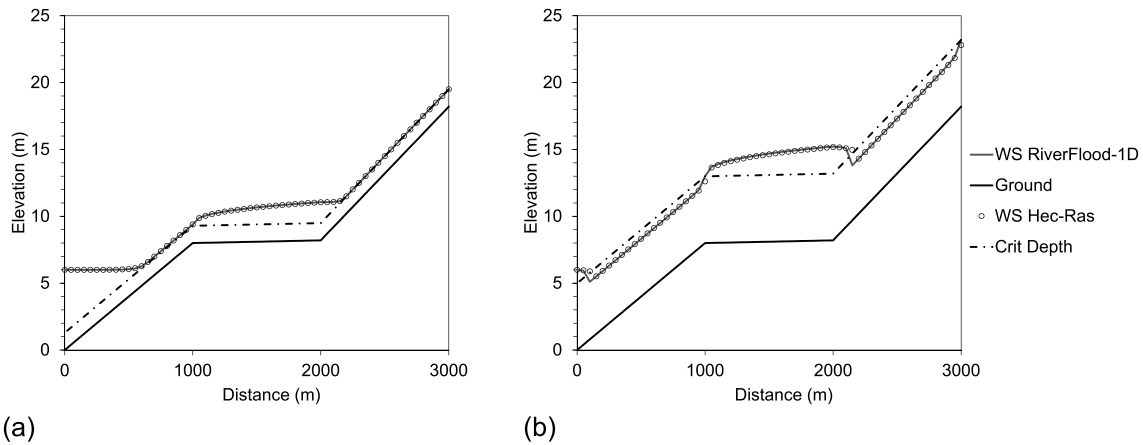


Fig. 7. Water surface profile (WS) computed by RiverFlood-1D and HEC-RAS for the test of steady water surface profile in compound channel with mixed flow regime for a discharge of (a) 1000 m³/s and (b) 8000 m³/s.

Table 2

Test 2 performance indexes computed against HEC-RAS simulation results.

Test case	MPE (–)	MAPE (–)	MaxAPE (–)	RMSE (m)
A	0.0003	0.0021	0.0080	0.0057
B	–0.0011	0.0129	0.2206	0.1938

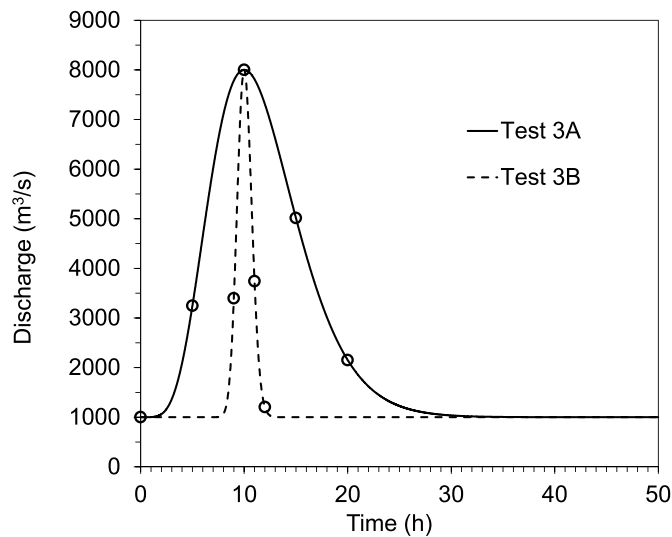


Fig. 8. Inflow discharge hydrographs used to test the RiverFlood-1D in simulating unsteady flow in compound channel with mixed flow regime (test 3). Circles mark the times of the results reported in Fig. 9.

Table 3

Test 3 inflow hydrograph parameters.

Test case	Q_b (m ³ /s)	Q_p (m ³ /s)	t_p (h)	γ (–)
A	1000	8000	10	1.17
B	1000	8000	10	1.005

proximity to the bed slope change. RiverFlood-1D water surface profile, instead, is characterized by an abrupt transition from supercritical and subcritical regimes, in agreement with the occurrence of hydraulic jump in the steady flow solution. This mechanism also explains the differences in Fig. 9 at distances of about 1000 m and 2000 m, which coincide with the two breaks in bed slope separating the three reaches of the synthetic channel (Fig. 6b). At these locations the flow changes regime, producing

a steep drawdown or a hydraulic jump in the steady-state solution; because the MC parameters in RiverFlood-1D are updated from that steady mixed-regime profile, the unsteady profile inherits the same abrupt transition, whereas HEC-RAS distributes the transition over a few cells and thereby smooths the surface and shifts its apparent position, consistently with findings of test 2. At $t = 10$ h the inflow is on the rising limb (Fig. 8), so discharge, celerity and jump position are all changing rapidly, and small differences in where each model locates the transition are amplified into the visible gap between the two profiles, even though the routed discharges remain in close agreement.

3.5. Test 4: Laboratory dam-break experiment on an initially dry bed

Dam-break flood is the unsteady free surface flow caused by a sudden collapse of a dam/barrier that blocks a mass of water that propagates downstream with a rapid variation of pressure and velocity in both time and space (Carrivick, 2010). Accurate simulation of the dam-break flow is challenging due to the high complexity of geometry involved, and flood propagation on an initially dry bed (Alamatian et al., 2021). In the equation of motion the inertia terms, compared with pressure, friction, and gravity terms, are important and should not be neglected. However, existing studies have shown how the parabolic approximation and MC model, even if they fail to reproduce some local phenomena, are suited to simulate dam-break flow (Prestininzi, 2008; Macchione, 1994).

The RiverFlood-1D model was tested by simulating flood routing of the Toce model, a dam break experiment recreated in the laboratory. The Toce model was designed and built at the ENEL-HYDRO laboratories in Milan to study the capabilities of different numerical models within the CADAM project (Soares Frazao and Testa, 1999; Caleffi et al., 2003). The physical model reproduces a 5 km reach of the Toce River located in the Northern Alps, at a scale of 1:100 (the total area of the experimental facility is 55 × 13 m) (Fig. 10). It is fully instrumented with stations that measure water level profiles at multiple locations with 1 s time interval. The model has a free overfall at the downstream end, and it is initially dry. The reach has several geometrical irregularities, that lead to the coexistence of subcritical and supercritical flow states. Topographic data are provided by 63 cross-sections. Based on the surface cover, the suggested Manning coefficient is 0.0162 s/m^{1/3}.

Flood routing simulation was undertaken with RiverFlood-1D model at 1 s interval (minimum time step value that can be set in the model in the current version) without any specific numerical stabilization procedures to manage model instabilities and initial dry bed condition. Upstream boundary condition discharge hydrograph is shown in Fig. 10b. Downstream boundary condition was set to critical depth. A simulation with HEC-RAS has been attempted with different parameter settings and model configurations, but all unsteady simulation runs

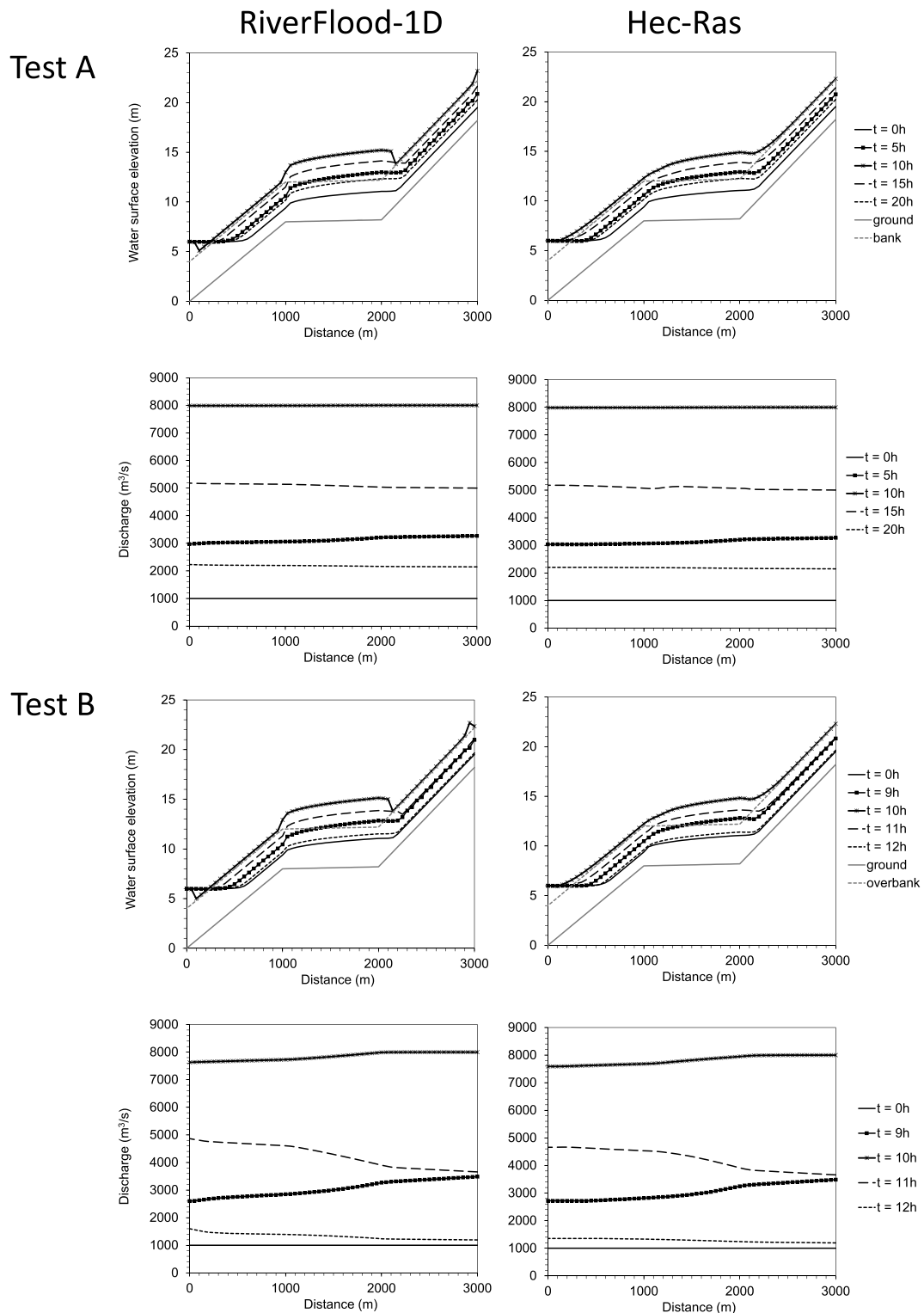


Fig. 9. Discharge and water surface elevation computed by RiverFlood-1D and HEC-RAS along the channel for given times for Test A and Test B.

terminated with error, so no comparison can be done with RiverFlood-1D model. While HEC-RAS was able to easily find a solution of test 3B for unsteady simulation with mixed regime, in a relatively simple geometrical setting, in the case of a very complex geometry and mixed flow like for the Toce model, the HEC-RAS unsteady solver went unstable. A geometry simplification, or cross-section interpolation, may help finding a solution, but we wanted to compare RiverFlood-1D and HEC-RAS with the same geometrical properties. It is acknowledged, in

line with common practice, that introducing interpolated cross-sections can stabilize the HEC-RAS unsteady solver at a modest additional computational cost. The comparison here was deliberately run on the unmodified 63-section geometry so that both models faced identical conditions. The relevant point is therefore that RiverFlood-1D produced a stable and reasonably accurate solution on the raw, highly irregular geometry without interpolation or solver-specific tuning.

The RiverFlood-1D computed water surface in P2, P5, P8, P18, P21,

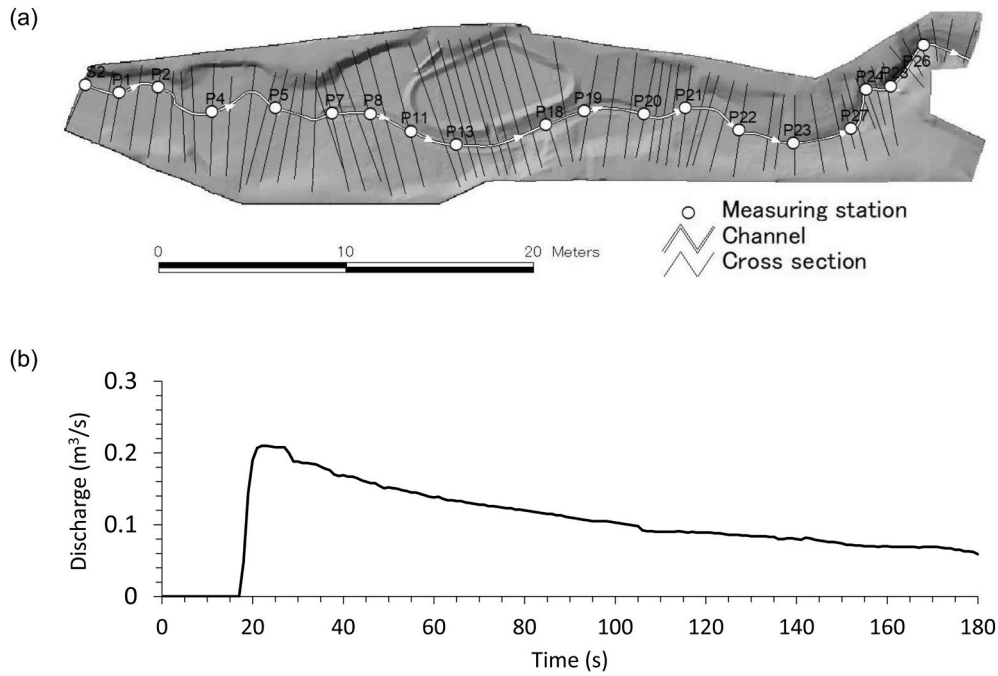


Fig. 10. (a) Topography of the Toce physical model, cross-sections, channel, and gauge stations position; (b) inlet discharge hydrograph.

and P24 stations are shown in Fig. 11. A delay in water surface rise is reported that increases in downstream sections, however the maximum elevation agrees with observations. The percentage error in simulating the maximum water depth is in the range $-0.14 - 0.069$ with an average error of -0.011 (Table 4). In Fig. 12, the water surface profiles along the main channel at 30, 60 and 120 s after the start of the experiment are reported. It is observed that after 30 s the waterfront has reached approximately 25% of the channel length, after 60 s it has reached approximately 60% and at 120 s water is present on the entire channel.

3.6. Test 5: Simulation of April 2009 river Po flood

In this section, the simulation of a flood event that occurred on the Po River is presented to demonstrate the models effectiveness in a real framework. With 652 km length, the Po is the longest river in Italy. The simulation concerns a 60 km long reach, starting at Isola Sant'Antonio,

Table 4

Observed and RiverFlood-1D simulated maximum water depth (m) and percentage error (PE) test 4.

Station	Observed (m)	Simulated (m)	PE (-)
P2	0.118	0.108	-0.087
P5	0.094	0.098	0.040
P8	0.145	0.125	-0.140
P18	0.148	0.157	0.064
P21	0.150	0.131	-0.129
P24	0.277	0.296	0.069

40 km upstream of the confluence of river Ticino, and ending at Pieve Porto Morone, 20 km downstream of the confluence with river Ticino (Fig. 13). Geometry data from 42 cross-sections are available, separated by variable distances ranging from about 92 to 2866 m. Interpolated

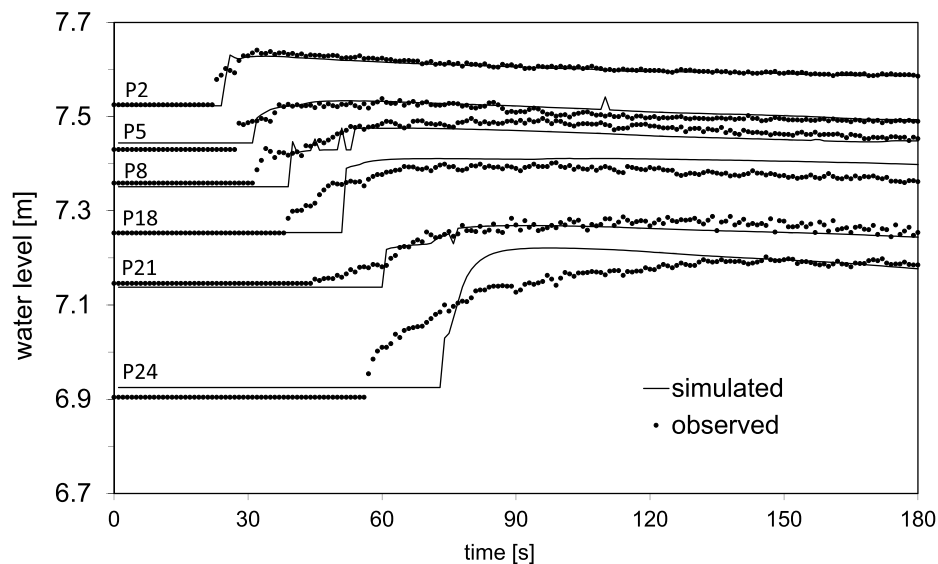


Fig. 11. RiverFlood-1D simulated hydrograph compared to corresponding measurements at stations P2, P5, P8, P18, P21, and P24.

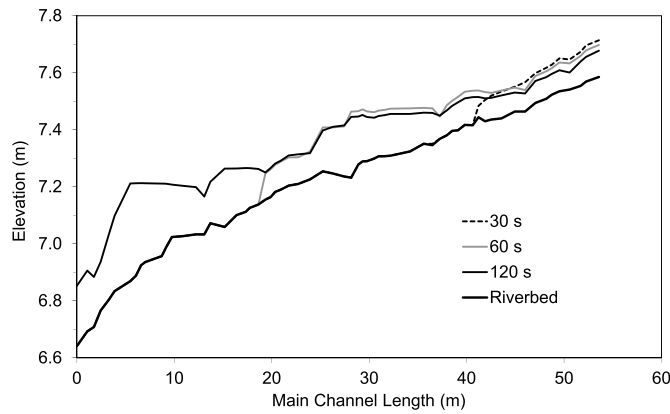


Fig. 12. Water surface profile along the Toce model at 30, 60 and 120 s after the dam break experiment start.

river cross-sections were added to keep a maximum distance between two cross-sections less than 1000 m. The Manning coefficient value was set equal to $0.03 \text{ s/m}^{1/3}$ for the main channel and $0.09 \text{ s/m}^{1/3}$ for the floodplain (Di Baldassarre and Montanari, 2009). The Manning roughness coefficient was not subjected to calibration, as this is not the objective of the present work. The upstream boundary condition discharge hydrograph is shown in Fig. 14a. The downstream boundary condition was set to normal depth.

The flood event hit mostly the Tanaro river basin, with little contribution in discharge from the right and left Po River tributaries in the investigated reach. A comparison between simulated and observed discharge and water depth at the Ponte Becca section is reported in Fig. 14. A good agreement is shown between RiverFlood-1D and HEC-RAS simulation results, with comparable performance indexes computed against observed values (Table 5). Discrepancies between simulations and observations can be explained by the contribution in the discharge of Po tributaries, which was small compared to the upstream basin contribution but not zero, and the uncertainty associated with discharge measurements in the Po River (Di Baldassarre and Montanari, 2009). A specific roughness coefficient calibration, for both HEC-RAS and RiverFlood-1D, or inclusion of dynamic roughness modelling due to vegetation (Herrera-Gómez et al., 2023, 2024) could decrease the lag time between observed and simulated peak discharge and maximum water depth.

3.7. Discussion

RiverFlood-1D represents a methodological advancement in the field of simplified flood routing by coupling, for the first time, the PAB scheme with the MC model within a unified computational framework. Previous implementations of the PAB scheme, starting from the original formulation by Todini and Bossi (1986) and its successors such as the Linearized Parabolic Model (Della Morte et al., 2001) and the ILILPM (Cimorelli et al., 2013), relied on the analytical solution of the linearized convective-diffusion equation for discharge routing. In RiverFlood-1D, the PAB framework is instead used to continuously update the steady water surface profile at each time step, providing the hydraulic parameters that are then fed into the MC routing equations. RiverFlood-1D inherits the well-known numerical robustness of the MC method, while the PAB steady-profile computation accounts for downstream boundary conditions and backwater effects that purely kinematic and diffusive approaches cannot represent.

The five benchmark tests span a wide range of hydraulic conditions, from subcritical to supercritical flow, from steady to highly transient regimes, and from idealized laboratory flumes to complex natural geometries (Table 1). The results collectively support the conclusion that RiverFlood-1D provides reliable predictions across this spectrum, though with nuances that deserve discussion.

Across the five tests, RiverFlood-1D reproduced the steady and unsteady benchmark results with accuracy comparable to HEC-RAS (Tables 2 and 5). For the Po River event the performance indexes of the two models are of the same order, with HEC-RAS marginally closer to the observed water depth (MAPE 0.080 against 0.119). The added value of RiverFlood-1D therefore does not lie in superior accuracy on cases that HEC-RAS already solves, but in three complementary aspects. First, robustness: in the Toce dam-break test (Test 4) RiverFlood-1D produced a stable solution on a complex, mixed-regime, initially dry geometry for which the HEC-RAS unsteady solver did not converge. Second, simplicity of coupling: the model is a compact Fortran 90 routine that reads standard HEC-RAS geometry, and is straightforward to embed as a routing module within hydrological and forecasting systems. Third, parsimony: by deriving its parameters from a steady mixed-regime profile, it extends the widely used MC approach to compound cross-sections with downstream control while retaining the stability and low data demand of hydrological routing. These features make the model well suited to operational, reach-by-reach real-time forecasting, a context in which full hydraulic solvers are more often used offline under expert supervision.

Regarding computational efficiency, in test 3 RiverFlood-1D

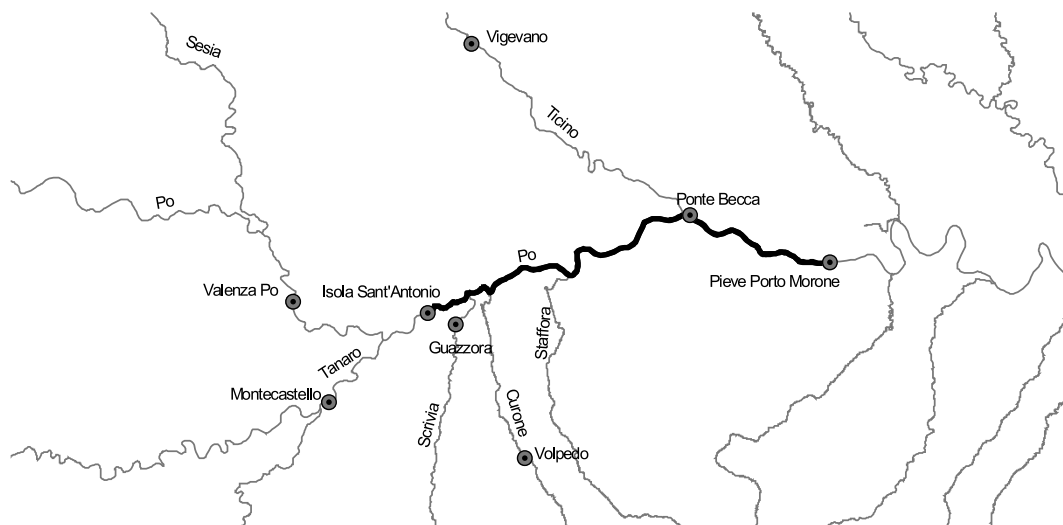


Fig. 13. Po river and tributaries, and river gauges location. Black line shows the Po reach simulated with RiverFlood-1D and HEC-RAS.

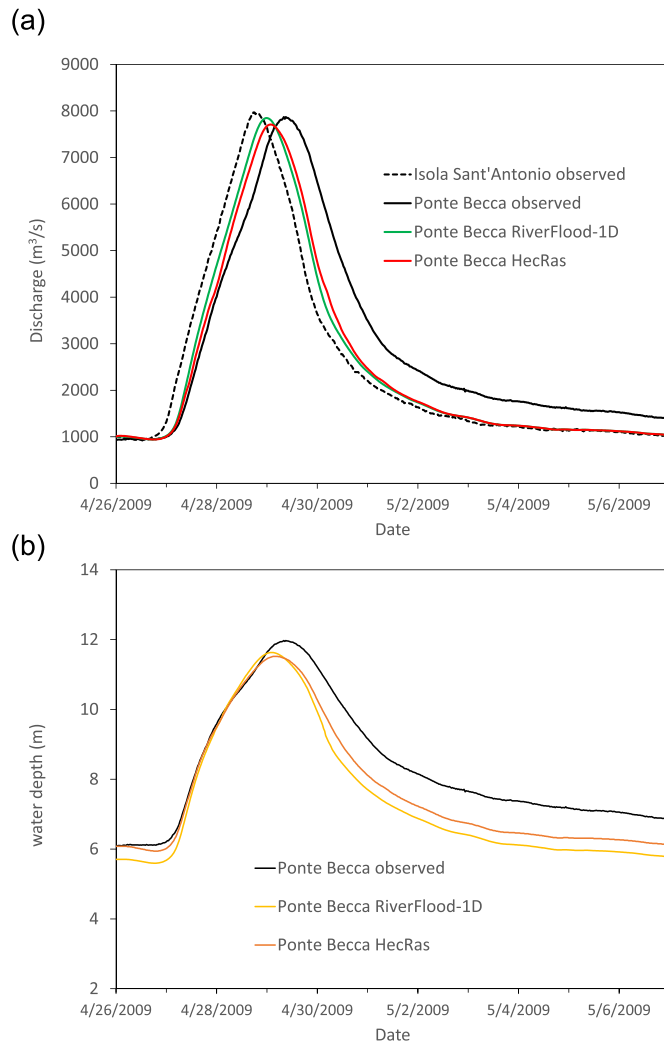


Fig. 14. (a) Upstream boundary condition discharge hydrograph (Isola Sant'Antonio observed) and observed and simulated discharge hydrographs at Ponte Becca, and (b) and water depth simulated by RiverFlood-1D and Hec-Ras models compared to observations at Ponte Becca station of the April 2009 flood.

Table 5

2009 flood simulation performance indexes of models RiverFlood-1D and HEC-RAS computed against observations.

Model	Discharge		Water depth	
	MPE (–)	MAPE (–)	MPE (–)	MAPE (–)
RiverFlood-1D	–0.163	0.230	–0.118	0.119
HEC-RAS	–0.169	0.210	–0.080	0.080

completed the 61 sections, 3000 time steps simulation in 2.1 s against 4.9 s for HEC-RAS on the same machine, that became 16.34 with the HEC-RAS option of data post processing. In test 4 RiverFlood-1D completed the 79 sections, 526 time steps Po River simulation in 2.6 s against 3.3 s for HEC-RAS, that became 5.8 with the HEC-RAS option of data post processing. Although HEC-RAS is already efficient, the simplified scheme avoids the iterative solution of the full momentum equation at every time step, and its single-routine structure removes the input/output and interface overhead incurred when an external hydraulic model is invoked repeatedly inside an operational chain. The HEC-RAS option for data post processing further increases the computational time required in case detailed information for all sections and all time steps are required.

For the Po River reach, a stable and fast routing tool of this kind can support real-time stage and discharge forecasting at gauged sections such as Ponte Becca and the propagation of upstream-basin forecasts toward downstream flood defences, complementing the more detailed but heavier two-dimensional studies used for floodplain mapping. The main limitations are intrinsic to the diffusion-wave assumption: the inertial terms are neglected, so strongly dynamic local phenomena are only approximated, as seen in the delayed water-surface rise in Test 4; lateral inflows and tributary contributions are not yet represented; the minimum time step is currently 1 s.

4. Conclusions

This study presented RiverFlood-1D, a one-dimensional simplified flood-routing model that couples the PAB scheme with a variable-parameter MC routing formulation. The principal novelty of this work is the coupling of the parabolic-and-backwater scheme with a variable-parameter MC routing in which the parameters are recomputed at every time step from a steady, mixed-regime water-surface profile in arbitrary compound cross-sections that accounts for the downstream boundary condition. To the authors' knowledge, this is the first time the PAB scheme has been built on the MC model in this way, generalizing hydrological routing to compound channels with backwater effects and flow-regime transitions.

The model was tested and validated against steady and unsteady problems in laboratory, synthetic, and real cases, by comparing RiverFlood-1D to HEC-RAS simulation results and observations. The RiverFlood-1D model proved to be accurate and stable for practical engineering applications. Its robustness and computational efficiency make the RiverFlood-1D model a good candidate for employment in real time flood warning systems.

The tests showed that the model reproduces steady compound-channel profiles and unsteady mixed-regime hydrographs with accuracy comparable to the full dynamic-wave solver HEC-RAS (water-depth MAPE of 0.0021–0.0129 in the synthetic tests and 0.119 in the real Po River event), that it resolves hydraulic jumps as sharp transitions consistent with the steady solution, and, most importantly for operational use, that it remained stable on the highly irregular Toce dam-break geometry where the HEC-RAS unsteady solver failed to converge.

The study also has limitations. The diffusion-wave approximation neglects the inertial terms, so rapidly varying local dynamics are only approximated, as reflected in the delayed water-surface rise simulated in the dam-break test; lateral and tributary inflows are not yet represented; the minimum time step is presently 1 s. The Po River validation was also limited to one flood event and one river reach; therefore, further applications are required before generalizing the performance of the model to other geomorphological and hydrological settings.

Coupled to a rainfall-runoff model, RiverFlood-1D can route forecast hydrographs within a real-time warning system, the setting in which its robustness and efficiency are most valuable.

Future model development will include bridges and vegetation roughness modelling, time step interval less than 1 s, and time step automatic adjustment. Future work will also address the representation of lateral and tributary inflows. The benchmark configuration, input and output files, and source code are openly available (Ravazzani, 2025) to support reproducibility and independent evaluation.

CRedit authorship contribution statement

Giovanni Ravazzani: Conceptualization, Investigation, Methodology, Software, Writing – original draft, Writing – review & editing. **Verónica Herrera-Gómez:** Data curation, Formal analysis, Investigation, Writing – review & editing. **Giuseppe Passoni:** Methodology, Resources. **Marco Mancini:** Supervision.

Declaration of competing interest

The authors declare that they have no conflict of interest.

Acknowledgements

Authors thank ARPA Piemonte for providing cross-section geometry, water level and discharge data of the Po River. Authors are grateful to Luca Di Stefano for help in performing analysis required for completion of his Master of Science's thesis. Many thanks to Alessandro Valiani and Valerio Caleffi for providing data of the Toce dam-break laboratory experiment.

Data availability

Name of software: RiverFlood-1D.
Developers: Giovanni Ravazzani. Contact: giovanni.ravazzani@polimi.it.
Year first available: 2025.
Operating System: Windows/Linux/macOS.
Required software: Fortran 90 compiler.
Program language: Fortran 90.
Source code at: <https://doi.org/10.5281/zenodo.15397505> (Ravazzani, 2025).

Configuration, input and output files of the benchmark tests are available at <https://doi.org/10.5281/zenodo.15397505> (Ravazzani, 2025). The executable for Windows OS and related .bat files to run the five tests are provided, as well. The executable has been compiled with Intel® Fortran Compiler – toolkit version: 2023.0.0, extension version 23.0.0070. Runtime Redistributable Libraries for Intel® oneAPI Compilers to run the executable on computers without Intel® Fortran Compiler installed, can be downloaded from: https://registrationcenter-download.intel.com/akdlm/IRC_NAS/19107/w_ifort_runtime_p_2023.0.0.25922.exe.

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